Vibration measurements on a modern steel bridge
Ward, H. S.

Publisher’s version / Version de l'éditeur:

Engineering Journal, 49, 9, pp. 15-19, 1966-10-01

NATIONAL RESEARCH COUNCIL

CANADA

CONSEIL NATIONAL DE RECHERCHES

VIBRATION MEASUREMENTS ON A MODERN STEEL BRIDGE

by

H. S. WARD

Reprinted from
THE ENGINEERING JOURNAL
Vol. 49, No. 9, September 1966,
p. 15-19

Research Paper No. 294
of the
Division of Building Research

OTTAWA
October 1966

Price 10 cents

NRC 9263

MESURE DES VIBRATIONS D'UN PONT MODERNE EN ACIER

SOMMAIRE

Le présent exposé décrit les méthodes de mesure des vibrations d'un pont d'acier à travées multiples, à poutres-maîtresses latérales, dont la longueur dépasse 2,000 pieds. Les mesures ont été réalisées à la travée centrale du pont, soumise à des charges dynamiques relativement élevées dues à l'impact des roues de camions roulant sur le tablier inachevé. Bien que les ouvriers travaillant sur le pont aient nettement ressenti les vibrations, les résultats des mesures montrent que les tensions induites dans les éléments du pont étaient faibles. Une méthode simple a été utilisée pour obliger la travée centrale à entrer en vibration sur ses fréquences propres, et on a obtenu une estimation de ses caractéristiques d'amortissement.

Vibration Measurements on a Modern Steel Bridge

Dr. H. S. Ward
Formerly Research Officer
Building Physics Section
Division of Building Research
National Research Council

EIC-66-BR STR 16

The ratio of live to dead load is generally higher for bridges than for most other structures. The major live load, that imposed by traffic, is dynamic in nature and this is generally accounted for in design by applying an impact factor to the static load. Because aesthetic considerations are leading to more slender structures, however, the time may be approaching when a more thorough structural dynamic analysis may be required in bridge design.

Accordingly the author was glad to avail himself of the opportunity to make a few vibration measurements during the final stages of construction of the MacDonald-Cartier Bridge Fig. 1. This bridge, which connects Ottawa to Hull, is of steel girder construction, with five spans and a total length of approximately 2000 feet.

Procedure

Vibration measurements were taken on two separate days, August 29 and October 13, 1965. On August 29, the concrete deck of the bridge was complete, but the final 1½ in. of bituminous material was not in place. This meant that the steel sections which act as part of the expansion joints in the deck were raised 1½ in. above the concrete deck; the expansion joints are at 45-ft. centres along the length of the bridge. At this time there was still much constructional activity, in particular with regard to the placing of the sidewalk safety fence. On October 13 the bridge deck was completed, and there was very little activity on the bridge.

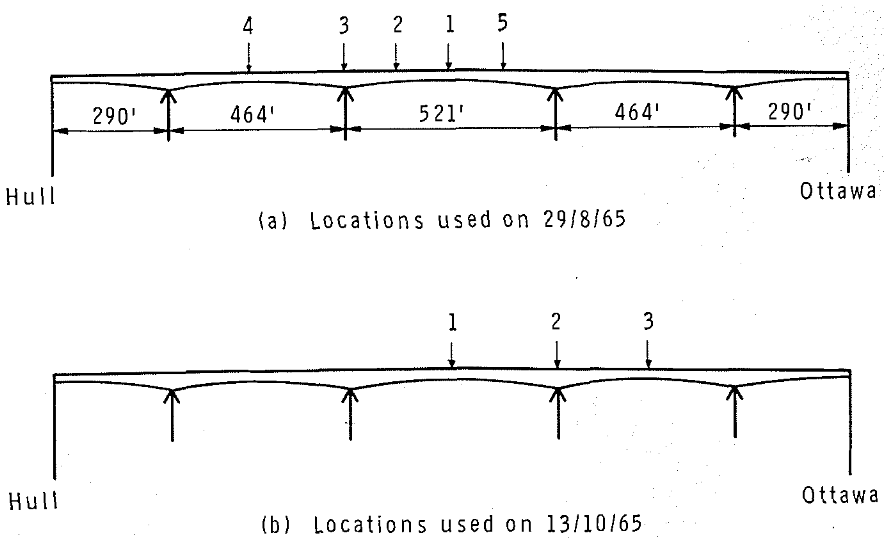
The locations at which the measurements were taken on the two dates are shown in Figs. 2a and 2b. In each case the measurements were taken on the eastern side of the bridge, on the traffic lane nearest to the central median. The mobile laboratory of the Division of Building Research, which weighs approximately 9 tons, was parked at the centre of the central span of the bridge so that it would not create an unsymmetrical loading condition. The vertical vibrations of the bridge were recorded with Mark II Willmore seismometers. These are velocity transducers with a sensitivity of 10 volts/in./sec., and they were adjusted to have a flat frequency response down to 0.3 cps.

Results

Several recordings were made on August 29 when vehicles were being driven across the bridge. The records shown in Fig. 3 were produced by a truck weighing approximately 10 tons moving at about 10 mph in the direction of Ottawa. This meant that the bridge was receiving a series of impacts due to the truck falling through 1½ in. every 3 seconds. Each series of impacts would be made up of the different sets of wheels going over the expansion joints, and the individual impacts would represents approximately 2 tons falling through 1½ in. because there were four sets of wheels. If it is assumed that the spacing

Fig. 1. View of the MacDonald-Cartier Bridge which connects Ottawa to Hull.

Fig. 2. Locations where vibrations were measured on the bridge



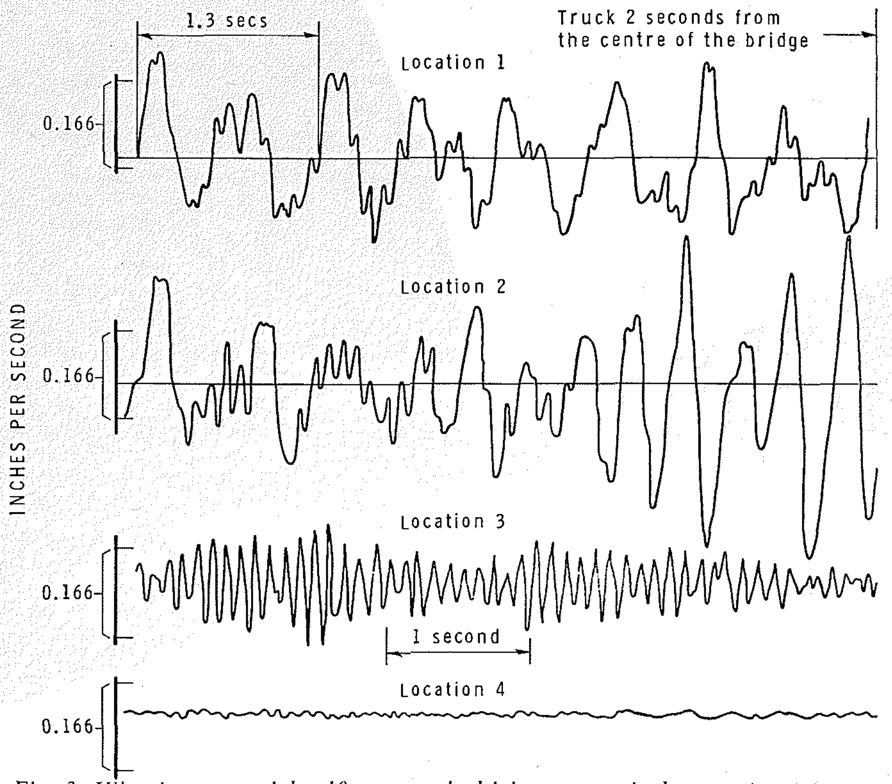


Fig. 3. Vibrations caused by 10 ton truck driving over raised expansion joints at approximately 10 M.P.H.

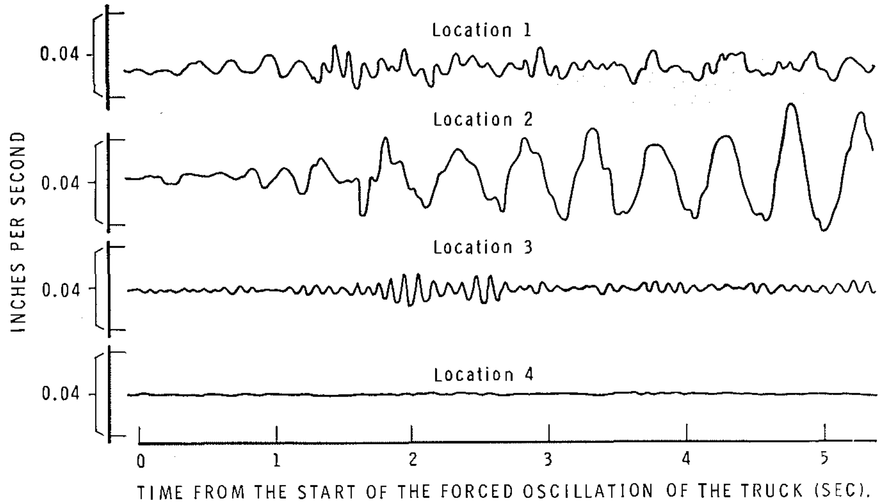


Fig. 4. Vibrations caused by the forced vertical oscillations of a stationary truck.

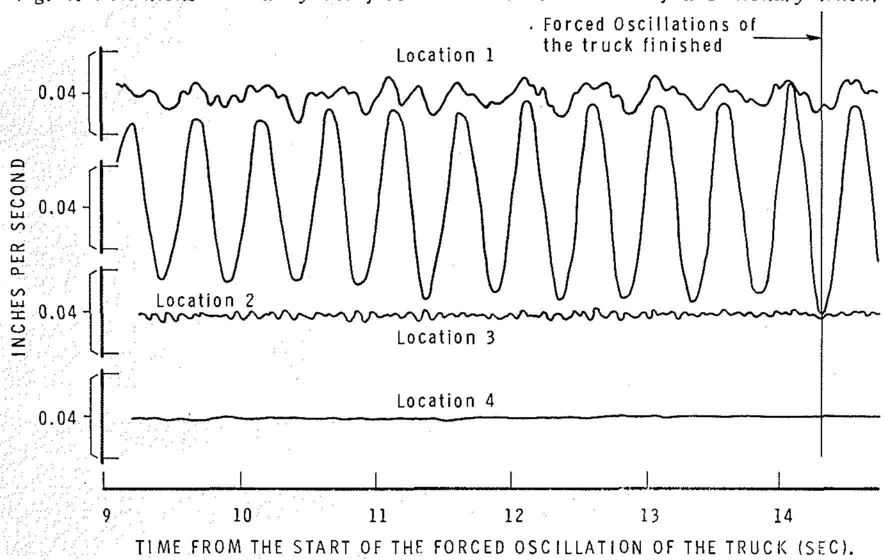


Fig. 5. Vibrations caused by the forced vertical oscillations of a stationary truck.

of the sets of wheels from front to back were 8 ft., 30 ft., and 3 ft., the time between impacts would be of the order of 0.5 seconds, 2.0 seconds and 0.2 seconds.

The records of Fig. 3 show that the response of the bridge to this complex forcing function is, naturally, itself complex. The records from left to right cover the time the truck was approximately 75 ft. to 30 ft. north of the centre of the bridge. Although the records are complex, there appears to be a natural mode of vibration of the central span with a period of 1.3 seconds or a frequency of 0.77 cps. This is demonstrated by the vertical oscillations at locations 1 and 2 which seem to consist of this low frequency vibration together with another vibration with approximately twice this frequency. The vibrations at the northern pier of the central span have a frequency of 10 cps, and the record for location 4 shows that very little vibration was transmitted from the central span to the adjacent span.

The amplitudes of vibration at locations 1 and 2 were relatively high and as the truck passed the vibrations were quite noticeable to anyone standing on the bridge. As the truck passed the centre of the bridge the highest vibration level obtained during the study was recorded; this level was 1 in./second peak to peak and the frequency was approximately 2 cps which would represent a displacement of 0.08 in. peak to peak or an acceleration of 12 in./sec^2 peak to peak. This acceleration is midway between the levels which, according to Reiher and Meister [1], humans find to be "perceptible" and "disturbing".

Another set of recordings was made of the response of the bridge due to the forced vertical oscillations of the stationary laboratory truck parked at the centre of the bridge. A visual display of the bridge vibrations was used to enable the people in the truck to force the vehicle to vibrate so that one of the natural frequencies of the central span was excited. Figures 4 and 5 show how the vibrations were built up and Fig. 6 shows how they decayed when the forced vibration of the truck was discontinued.

The mode of vibration of the bridge that was excited had a frequency of 2 cps. The reason it was possible to excite this particular mode can be explained by considering the vibration characteristics of the truck and its suspension. For the case of vertically applied loads the vehicle can be considered equivalent to a heavily damped single-degree-of-freedom system. This was demonstrated by recording the vibrations of the vehicle caused by a person jumping up and down at different rates. It was shown in this manner that the truck could be forced to vibrate through a frequency range from 2 cps to 5 cps.

The records demonstrated that at 2 cps the bridge had a maximum amplitude at the quarter point of the central span, and again there was hardly any transmission of vibration to adjacent spans. The maximum displacement caused by this type of excitation was 0.009 in. peak to peak. An estimate of the amount of damping in the central span of the bridge when it was completed except for the bituminous cover can be determined from Fig. 6 by using the ratio of successive amplitudes of vibration. This method gives an estimate of 2% of critical damping for the mode of vibration that was excited.

In order to obtain some idea of the mode shape of the central span with a frequency of 2 cps an extra transducer was placed at location 5 on August 29 and the span was again forced to vibrate by using the forced vibrations of the truck. The results are shown in Fig. 7 where it can be seen that there was little motion at the centre of the bridge, at the northern pier of the central span, or at the centre of the adjacent northern span. A large amplitude of vibration was excited at the quarter points of the spans, and the interesting thing to notice is that as the one point moves down the other one moves up. This behavior leads to the conclusion that the second mode of vibration of the central span of the bridge was excited by the forced oscillations of the truck. It also indicates that the truck was not located exacly at the centre of the bridge because in this case it would have been impossible to excite this asymmetrical mode.

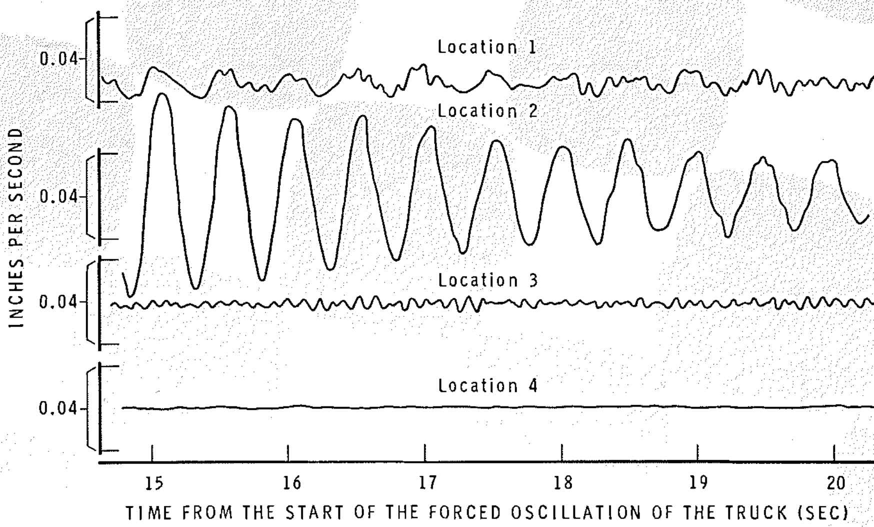


Fig. 6. Vibrations caused by the forced vertical oscillations of a stationary truck.

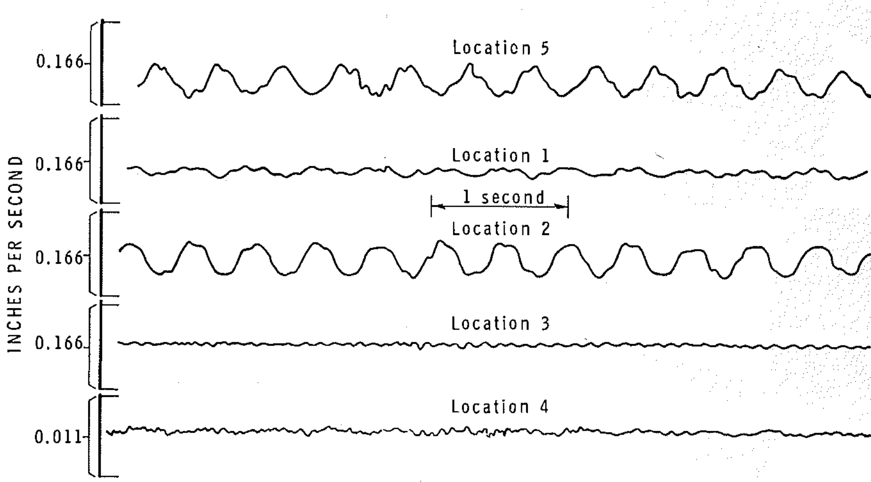


Fig. 7. Vibrations at five locations on the bridge caused by the forced vertical oscillations of a stationary truck.

Theoretical Analysis

The information obtained to this point indicated that for the types of load investigated the central span behaved as an individual unit since negligible motion was transmitted to adjacent spans. The indications were that the fundamental frequency of the span was around 0.77 cps and the second mode frequency was around 2 cps. Following these observations, it seemed worthwhile to investigate what answers a theoretical analysis might give for these frequencies. A greatly simplified analysis of the central span of the MacDonald-Cartier Bridge was therefore performed. The assumptions that were made are shown in Fig. 8a. It was assumed that the span was fixed at each end and that the central two thirds of the span had a second moment of area equal I/4 whereas the rest of the span had a value equal to I. The numerical values of I were obtained by taking average values from the structural plans of the bridge. The total load acting on each girder of the bridge was assumed to be 6000 kips.

The flexibility matrix of the girder was calculated by using the Moment-Area method to calculated the deflections, at six equally spaced intervals across the span, due to a unit load placed at each of these points in turn. It was then assumed that one sixth of the weight supported by the girder was concentrated at these points and the Stodala method was used to calculate the frequencies and shapes of the first two modes. The equations for the frequencies of the fundamental mode, f_1, and the second mode, f_2, were as follows.

$$f_1 = \frac{6.80}{2\pi}\sqrt{\frac{6EIg}{WL^3}} \qquad (1a)$$

$$f_2 = \frac{16.91}{2\pi}\sqrt{\frac{6EIg}{WL^3}}. \qquad (1b)$$

In these equations E represents Young's modulus of steel and g is the acceleration due to gravity. Substitution of the appropriate values in Eqs. (1a) and (1b) led to values of 0.87 cps and 2.16 cps for f_1 and f_2 respectively. The mode shapes are also shown in Figs. 8b and 8c. It would seem that this very simplified theoretical approach leads to good agreement with the information obtained from the vibration measurements.

Two further records of interest were taken on August 29. Figure 9 shows the effect of unloading from a truck one section of the pedestrian safety fence close to location 4. The maximum vibration level at this point was around 0.08 in./second peak to peak and as the record shows, it was a high frequency vibration. The records also show that the vibration was not transmitted to the central span. Finally, Fig. 10 shows the background level of vibrations when there was little or no activity on the bridge. Under these conditions, the principal vibrations were at 8 cps and the amplitude levels were of the order 0.01 in./second peak to peak.

On October 13 the bridge was completed and the opportunity was taken to determine the effect of a vehicle being driven across the bridge at different speeds. The mobile laboratory was parked at the centre of the bridge

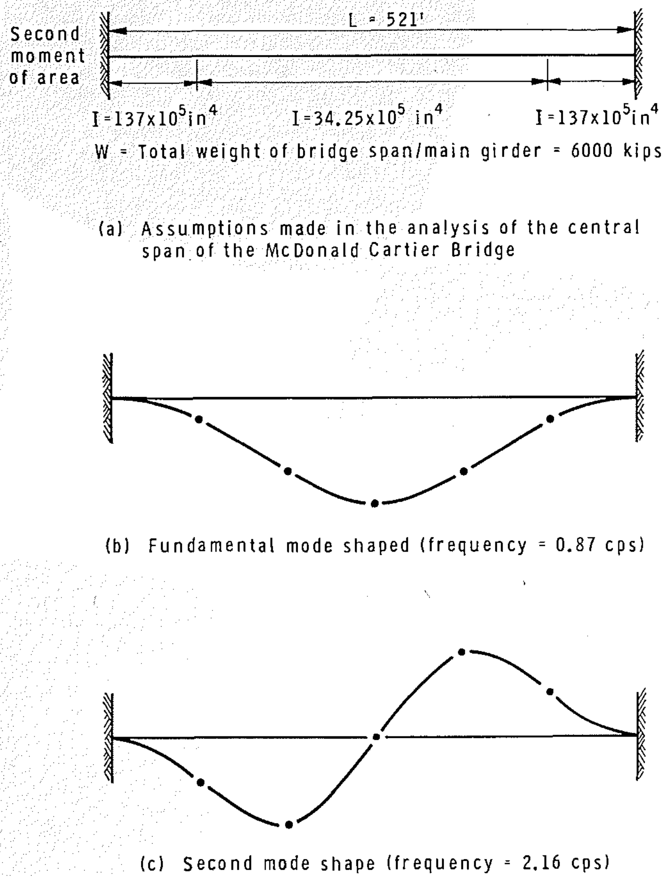


Fig. 8. Simplified analysis of the central span of the MacDonald-Cartier Bridge.

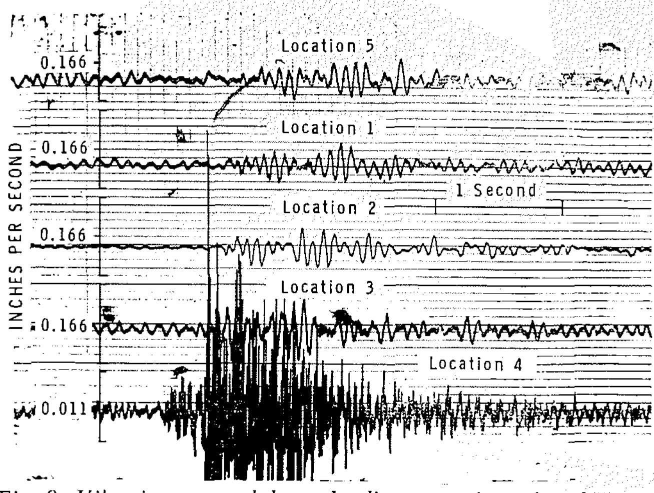


Fig. 9. Vibrations caused by unloading a section of pedestrian safety fence.

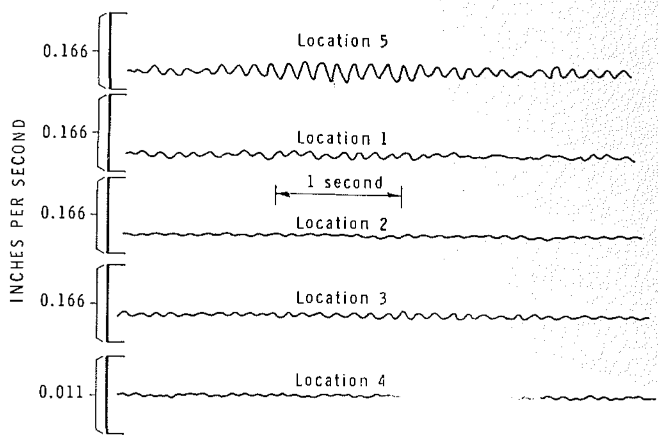


Fig. 10. Background vibrations on the bridge.

and a 1½-ton panel truck was driven across the bridge. Figures 11 and 12 show the vibration records obtained at three locations when the panel truck was being driven northwards at a speed of 30 mph. The records show that the amplitude of vibration was a maximum at each location as the truck passed tht location, and this level was around 0.02 in./second, peak to peak. The frequencies of the vibrations were quite high, between 10 to 15 cps, although there does appear to be a low frequency component for location 1 in Fig. 12 (shown by the dotted line).

If a frequency of 10 cps is used, then the maximum displacements caused by the panel truck were in the region of 0.0003 in. peak to peak. If the low frequency vibration is as sketched in Fig. 12 it would represent a displacement at the centre of the central span equal to 0.003 in. peak to peak. When this level is increased by a factor of six, to make it equivalent to the effect of say a 10-ton truck, it is still less by nearly a factor of 4 than those shown in Fig. 3. Table I shows the vibration levels that were caused by the panel truck being driven at four different speeds. The amplitudes and frequencies of vibration were essentially the same in each case.

Table I

Vibration Levels Measured on the MacDonald-Cartier Bridge due to the Passage of a 1½-Ton Truck

Truck Location	*Instrument Location*	5	15	20	30
		Truck Speed, m.p.h. (*peak-to-peak velocity, in./sec*)			
1	1	0.020	0.030	0.016	0.020
	2	0	0.012	0	0
	3	0.004	0.010	0.008	0.004
2	1	0.008	0.016	0.012	0.008
	2	0.020	0.014	0.018	0.020
	3	0.020	0.006	0.010	0.020
3	1	0.008	0.006	0.006	0.008
	2	0.004	0	0	0.004
	3	0.024	0.012	0.018	0.024

Discussion of Results

The results obtained on the two days show the important influence of the type of exciting force on the response of a structure. When there was a definite pronounced impact on the bridge, caused by vehicles moving across the raised expansion joints, relatively high vibration levels were created in what appeared to be the first two modes of vibration of the central span. When the bituminous coating was in place there was no longer a pronounced impact caused by vehicles; the amplitudes of vibration were much smaller and a mode of vibration higher than the second was excited (perhaps the fourth mode).

The results obtained for the response of the central span to the forced oscillations of a stationary truck indicate that there was only 2% of critical damping for the second mode of vibration, when there was no bituminous material in place. It would have been of great interest to see if the addition of bituminous material had increased the damping, but unfortunately this was overlooked at the time.

The most important factor from the

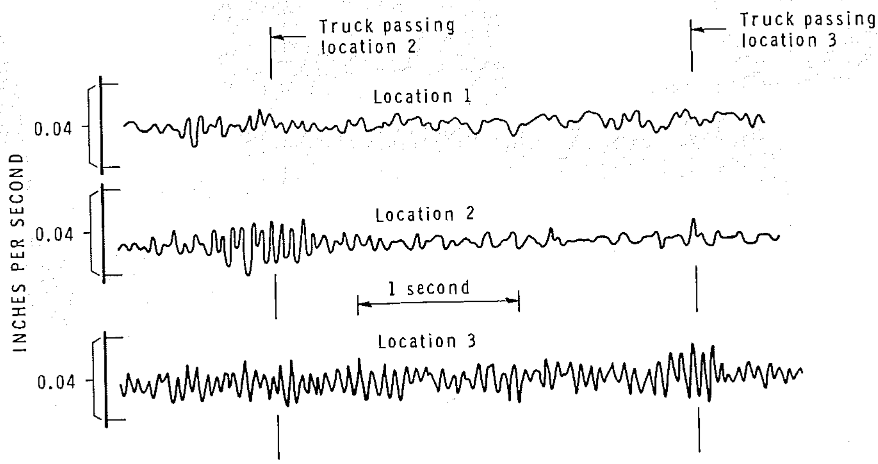


Fig. 11. Sample of vibration records caused by 1½ ton truck travelling at 30 M.P.H.

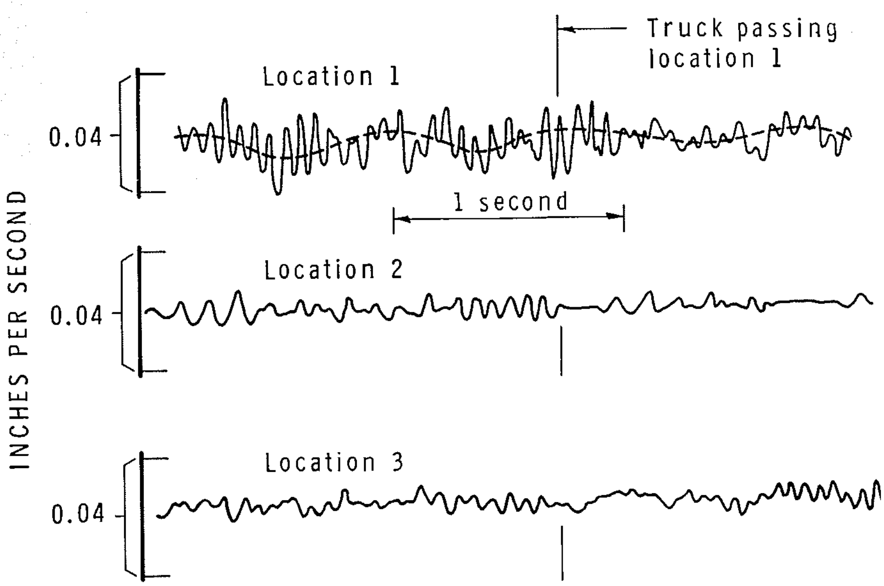


Fig. 12. Sample of vibration records caused by 1½ ton truck travelling at 30 M.P.H.

designer's viewpoint will be the stresses caused by the dynamic loads. Since the simple model shown in Fig. 8 gives results which are in agreement with the measured values, it is possible to make an estimate of the stresses induced by the vibrations that are described here. For example, the 10-ton truck moving across the central span caused a maximum vibration of 0.08 in. peak to peak at the quarter point of the span at a frequency of 2 cps. This frequency represented the second mode shape which is shown in Fig. 8c. A crude estimate has been made of the second differential of the curve in Fig. 8c when the deflection at the quarter point is 0.04 in. In this case the maximum stress, σ, occurs at $L/3$ from the supports and its value is given by,

$$\sigma = \frac{Md}{I} = Ed\frac{d^2y}{dx_2} \qquad (2)$$

where d is the distance from the neutral axis to the top of the flange of the girders.

If E is taken to equal 30 x 10^6 lb./sq. in., the maximum stress, caused by the highest vibration level measured during the two days, is found to be 250 lb./sq. in. This value is based on the assumption that the deck and the girder act in an integral manner.

Even though the vibration levels recorded when a single vehicle moved across the completed bridge were low, there is a good possibility that under full traffic loading the vibration levels could become comparatively large. It would therefore be of interest to try and measure the vibrations under these conditions since it might be of significant interest to designers. Far more information could have been obtained if there had been a controlled method of exciting the bridge. Some thought was given to different ways of doing this, but the great difficulty with large structures like the MacDonald-Cartier Bridge is the fact that a large force is required at a low frequency. If any more work is to be done in this field, it would be worthwhile to give serious thought to this factor.

Conclusions

The levels of vibration measured before the final road surface was in place were very noticeable to people on the bridge, but the instrumental records showed that, beyond the point of impact itself, vibrational stresses induced in the bridge were probably no greater than 250 psi. When the final bituminous surface was in place the cause of the impacts was eliminated, and the vibration levels created by road traffic were significantly reduced.

The vibration measurements indicated that the bridge was lightly damped and acted as if the separate spans were fixed at the supports. The close agreement between the measured frequencies of vibration and those obtained from a simple theoretical model indicates that this twofold approach to a study of the dynamic response characteristics of modern bridges could be a fruitful field of research.

Acknowledgements

The author is grateful to two members of the Dominion Bridge Company, Ramsey Murray who first suggested the project and Dr. Ross Chamberlain whose comments on an early draft of the paper were helpful in determining its final form. Thanks are due also to the structural engineering staff of the Department of Public Works, Canada, who reviewed the paper and concur in its publication. This paper is a contribution from the Division of Building Research, National Research Council, Canada, and is published with the approval of the Director of the Division.

References

1. **Die Empfindlichkeit des Menschen gegen Erschutterungen.** H. Reiher and F. J. Meister, Forschung auf dem Gebiete des Ingenieurwesens, 1931, 2(11), p. 381.

PRINTED IN CANADA